3.11 GEOLOGICAL RESOURCES/STRUCTURAL INTEGRITY REVIEW

3.11.1 Introduction

The Shore marine terminal wharf is located in the northeast portion of Bulls Head Channel, approximately 1 mile east of the Benicia-Martinez Bridge. Because the wharf is located in protected waters, the primary design loads for its stability are the berthing and mooring loads from tankers, and potential effects of earthquake forces and displacements on the structure.

The wharf is located in the seismically active San Francisco Bay Area. Moderate to severe earthquakes on any of the numerous faults in the area could impact the site. Of particular concern is the Concord/Green Valley Fault, which is located approximately 1,000 to 1,500 feet east of the site. The active Concord/Green Valley Fault is capable of producing an earthquake with a moment magnitude (Mw) of about 6.9.

3.11.2 Existing Conditions

3.11.2.1 Geotechnical Considerations

Geology

Regional Geology

California is located on the boundary between the Pacific and North America tectonic plates. The Pacific plate comprises the entire northern Pacific Ocean and the North America plate includes the remainder of North America and western half of the Atlantic Ocean. The North America plate is drifting southwesterly relative to the Pacific plate and overriding it. The main line of contact between these two plates is the San Andreas Fault system.

San Francisco Bay area lies within a geologically very active and dynamic part of the Coast Ranges geomorphic province of California, which is characterized by a series of nearly parallel mountain ranges (Goldman 1969). Active faults, including the Concord/ Green Valley, West Napa, Calaveras, Hayward, San Gregorio, and San Andreas Faults, are roughly parallel the western and eastern limits of the Bay. The Bay began forming during the Pleistocene Epoch, approximately 2 million years ago, when the San Francisco-Marin block began to tilt eastward along the Hayward Fault. The eastern side of the block became a depression and filled with sediment and water.

The bedrock units underlying the area east of the Hayward Fault range from Jurassic-Cretaceous to Quaternary age (approximately 135 million years old to recent). The oldest unit is called the Franciscan Formation. This formation probably originated on the Pacific Ocean floor and was welded to the western margin of the American continent by plate movement. Subsequently, it was pushed upward through the younger sedimentary rock to form the backbone of the Diablo Range (Contra Costa County 1975). The strata of this bedrock formation are highly distorted and partially metamorphosed through heat and compression. The Franciscan Formation primarily consists of interbedded sandstone and shale, limestone, radiolarian chert, and metavolcanic rocks (Goldman 1969).

The next oldest bedrock formation in Contra Costa County is the Great Valley Sequence, a thick sequence of Tertiary age sandstones and shales that overlies the Franciscan Formation. The Great Valley Sequence is sedimentary rock formed under ancient seas that once existed on the American continent. The youngest consolidated (hard) rock is the group laid down during the geologic age known as the "Tertiary." This unit is largely sedimentary rocks not yet hardened as have the older units. The youngest surface formations are the deposits of Quaternary-age marine sediments known as "bay mud." Figure 3.11-1 depicts the regional surface geologic conditions of the Suisun Bay and Carquinez Straits region near the project site.

Site-Specific Geology

Site-specific characteristics of the underlying geologic conditions described in this section are based on the regional studies of the Bay conducted by the California Division of Mines and Geology (CDMG) (Goldman 1969; Treasher 1963), and site-specific geotechnical investigations conducted during the development of the Shore terminal facilities (e.g., Woodward-Lundgren & Associates 1973a,b).

At the Shore marine terminal site, the local surface conditions range from rock outcrops near the southern portion of the Shore Terminals' complex, to the marsh/wetland deposits of the central area, to the sediments beneath Suisun Bay. The sediments that overlie the bedrock (described in the previous section) consist of Pleistocene alluvium and late Quaternary-age (Holocene) bay mud. During late Pleistocene and Holocene time, the sea level fluctuated several times. Lower elevation areas were exposed and subsequently submerged during changes in sea level, allowing for the deposition of the young bay mud. Young bay mud is of Holocene age (less than about 11,000 years old), and consists of gray silty clay typically very soft-to-soft in the upper portions of the profile and semiconsolidated (firmer/stiffer) in the lower portions.

Goldman's (1969) contour maps of the top of bedrock suggested that the top of bedrock lies approximately 50 to 150 feet below MLLW near the wharf and trestle location. A geotechnical study by Dames & Moore (1955) revealed that the bedrock surface generally slopes downward from the rock outcrops just south of the Shore marine terminal complex, to approximately Elevation –40 to –60 feet just south of the shoreline. This study did not extend as far as the wharf and trestle location. The overlying sediments described in the Dames & Moore (1955) study are congruent with the general geology described by Goldman (1969) and others. These included interbedded Pleistocene alluvium (silts, sands, and clays) overlain by more recent marsh deposits (peat and other organic deposits) and young bay mud.

3.11-1 – Surface Geology

Six borings drilled by Woodward-Lundgren (1973a,b) for the wharf and trestle provided more detail of the subsurface conditions. Three borings were drilled along the wharf alignment, and three were drilled along the trestle alignment. The deepest boring was drilled to Elevation –110 feet (MLLW).

Woodward-Lundgren (1973a) described the site and soil conditions as follows: "Based on the results of the exploratory borings, the bay bottom along the proposed wharf alignment slopes downward toward the east from Elevation –22 to Elevation –30 MLLW. The bay bottom surface along the trestle alignment slopes up toward the shoreline from Elevation –30 to Elevation 0 MLLW."

"The trestle alignment and the eastern half of the wharf alignment are overlain with a 10- to 18-feet-thick deposit of soft gray silty clay (bay mud). Soils encountered below the soft bay mud deposit and exposed over the western half of the wharf site consist of layers of very stiff clay and medium dense to dense sand and silt."

"Two strata of medium dense to dense sand were encountered over most of the offshore site. The upper sand stratum, which ranges roughly between Elevations –40 and –60 MLLW, is not continuous. The layer disappears at the east end of the wharf and near the south end of the trestle. The second and deeper layer is encountered at about Elevation –84 MLLW and appears to be continuous. This strata is composed of dense to very dense gravelly sand."

Bedrock was not encountered in these borings and is thus expected to be located deeper than Elevation –100 feet.

Figures 3.11-2, 3, and 4 show the boring locations, cross section along the wharf, and cross section along the trestle, respectively.

Site-specific geotechnical data help clarify the site conditions and associated considerations in several respects:

- Along the wharf alignment, there would have been little (approximately 0 to 5 feet thickness) young bay mud after the berth area was dredged to Elevation –35 feet (although there could have been some sedimentation from the time the wharf was constructed). Thus, lateral pile capacity would not be governed as much by the young bay mud, versus a case where the mud was assumed (due to lack of site-specific data) to be several tens of feet thick.

- Along most of the wharf alignment, sand layers between about Elevation –40 and –60 feet could potentially reduce lateral pile capacity and increase downdrag forces should liquefaction of a portion of the sand occur during seismic shaking. Woodward Lundgren characterized this sand as medium dense to dense based on the visual descriptions, probable geologic age (Pleistocene), and limited blow count and soil unit weight information. The sampling tools and protocols used during the exploration program, however, did not include the standard penetration test (SPT), an industry standard for evaluating liquefaction potential.

3.11-2 – Test Boring Locations

3.11-3 – Soil Profile at Wharf

3.11-4 – Soil Profile at Trestle

- At locations along the wharf with little or no young bay mud covering the top of the sand layer, lateral spreading of the uppermost portion of the sand could occur during seismic shaking if the sand layer surface slopes appreciably (for example due to dredging or scouring near the piles) and the sand is not dense enough. Lateral spreading would induce additional lateral forces on piles (in addition to the loss of lateral capacity mentioned above).

- Lateral behavior of most of the trestle piles is governed by the presence of the young bay mud. If liquefaction of medium dense sands between Elevation –40 and –60 feet occurred during seismic shaking, it could affect the axial capacity of the trestle piles, but would have less impact on lateral capacity.

- The variation in soil conditions across this fairly large site could result in spatial variations in the magnitude and timing of ground response during seismic shaking, which could then result in similar variations in the structural response of the wharf and trestle.

Seismicity

Regional Seismicity

The San Francisco Bay region lies along a major, seismically active plate boundary. The San Andreas Fault, which forms the boundary between the Pacific and North America tectonic plates, has produced numerous earthquakes in the Bay Area during historic and prehistoric times. Movement between the plates has created several other active faults parallel to the San Andreas, including the Hayward, Calaveras, Greenville, Concord/Green Valley, Rodgers Creek, and San Gregorio Faults. These faults create a zone of faulting approximately 50 miles wide through the greater San Francisco Bay Area. These faults and other faults that are close to the site are shown on Figure 3.11-5. The approximate distance from the site, estimated moment Mw for earthquakes along the faults, and slip rates of the faults are summarized in Table 3.11-1.

Several major historic earthquakes have occurred within the Bay Area on several of the major faults. A major earthquake occurred in 1836 and 1868 along the Hayward Fault, which is located approximately 13.5 miles from the site. Both earthquakes have estimated magnitudes of around Mw = 7. Slow movement, or creep, along the Hayward Fault is estimated to be approximately 9 millimeters per year (mm/yr) for a portion of the southern segment and around 5 mm/yr for the northern segment (Lienkaemper and Borchardt 1996). Joint studies by the CDMG and the U.S. Geological Survey (USGS) indicate that an earthquake with a maximum Mw of 7.1 could occur along the Hayward Fault (Petersen et al. 1996). In addition, the Working Group on California Earthquake Probabilities (1990) estimates there is a 28 percent chance that a magnitude 7 or greater earthquake will occur on the northern segment of the Hayward Fault within the next 30 years.

3.11-5 – Regional Fault Map

Table 3.11-1
Known Active Faults in Site Vicinity

Fault	Approximate Distance from Site (miles)	Estimated Maximum Magnitude (Mw)	Slip Rate (mm/year)	Recurrence Interval (years)
Concord/Green Valley	0.8	6.9	6	176
Greenville	9.4	6.9	2	521
West Napa	11.6	6.5	1	701
Calaveras (north)	13.0	6.8	6	146
Hayward	13.5	7.1	9	167
Rodgers Creek	13.5	7.0	9	222
Great Valley (seg. 4 to 6)	16.2 to 20.6	6.5 to 6.7	1.5	472 to 622
Hunting Creek	28.7	6.9	6	194
San Andreas (1906)	31.2	7.9	24	210
San Gregorio	32.9	7.3	5	400
Point Reyes	40.6	6.8	0.3	3503
Monte Vista	42.3	6.8	0.4	2410
Calaveras (south)	44.9	6.2	15	33
Maacama (south)	48.7	6.9	9	220
Note: Fault parameters were adapted from the CDMG and USGS fault database, Anderson et al. (1996)				

Another major earthquake occurred in 1861 on the Calaveras fault, which is located approximately 13 miles south of the site. This earthquake caused surface rupture for 8 miles through San Ramon Valley and caused severe damage within Contra Costa County. The major earthquakes on San Andreas Fault can cause significant ground shaking with high potential for damage to structures. The 1838, 1906 (estimated Mw = 8), and 1989 (Mw = 7.1) earthquakes on the San Andreas Fault are the major earthquakes that have occurred in the past 200 years. The 1906 and 1989 (Loma Prieta) earthquakes caused major damage to structures in the Bay Area. Estimated Mws of future earthquakes for various strands of the San Andreas in the Bay Area vary from Mw 7.0 to 7.9 (Peterson et al. 1996). The "Mare Island" earthquake of 1898, along the southern end of the Rodgers Creek Fault, which is approximately 13.5 miles from the Shore terminal, is also of historic significance. Toppozada et al. (1996) believe that the earthquake epicenter was located near the southern end of the fault, and its estimated Mw was 6.2.

Site-Specific Seismicity

The Shore terminal is surrounded by Concord/Green Valley Fault on the east, West Napa and Rodgers Creek faults on the northwest, Hayward fault on the west, and Calaveras fault on the south as shown on Figure 3.11-5.

The Concord/Green Valley fault is located less than 1 mile away from the site and is believed to be able to produce an Mw 6.9 earthquake every 176 years. Although in the 150-year history no major earthquake is recorded in this fault, most of the Working Group on Northern California Earthquake Potential (1996) felt that the Concord/Green Valley fault zone is likely to rupture in one major event. There was concern that the ruptures might occur beneath Suisun Bay.

The next nearest faults are the Greenville and West Napa faults, whose slip rates are in the order of 2 and 1 mm/year, respectively. Although the slip rates for these faults are relatively low, it is believed that they can produce large earthquakes with a range of Mw 6.5 to 6.9.

Active faults, as defined by the State Board of Mining and Geology (Hart and Bryant 1997), do not transect the Shore terminal. An active fault, as defined in the Alquist-Priolo Earthquake Fault Zoning Act, is one that has had surface displacement within Holocene time (about the last 11,000 years). The purpose of the Alquist-Priolo Act is to regulate development near active faults to mitigate the hazard of surface rupture (Hart and Bryant 1997).

Several inactive faults or pre-quaternary active faults (over 2 million years old), including the Southampton and Franklin Faults, are within several miles of the site. The Southampton Fault is located approximately 4 miles west of the site, and Franklin Fault is located approximately 6 miles west of the site. The Franklin Fault is believed to be the northern extension of the active Calaveras Fault.

Caltrans (1996) developed the Caltrans California Seismic Hazard Map that shows contours of peak acceleration. These contours reflect Maximum Credible Events (MCEs) for the various contributing faults, and apply to ground motion for rock or stiff soil. In the vicinity of the Shore marine terminal, the map shows a peak acceleration contour of 0.5 g which implies that strong ground shaking is likely should a major earthquake on a nearby active fault occur.

<u>Tsunamis</u>

Tsunamis are sea waves created by undersea fault movement. They are long-period sea waves produced by any large-scale, short-duration disturbance on the ocean floor, such as shallow earthquakes, volcanic eruptions, or landslides. When the waveform reaches the coastline, however, it pushes upward from the ocean bottom to create a high swell of water that breaks and washes inland with great force. Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned several people in Crescent City, California.

A tsunami originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. The runup for the 100-year tsunami near the Golden Gate is 10 feet (Ritter and Dupre 1972), which may be regarded as a reasonable maximum for future events. The available data indicate a systematic diminishment of the wave height from the Golden Gate to the head of the Carquinez Strait and on into Suisun Bay. The proposed MOTEMS Section 3.5.7 provides estimated tsunami run-up for areas of California. The maximum expected increment of wave height near the Shore terminal for the 100-year return period event is estimated at 3.3 feet, and for the 500-year return period event is estimated at 4.0 feet.

3.11.2.2 Structural Condition of the Shore Marine Terminal

Description of the Marine Terminal

The Shore marine terminal consists of a 1,745-foot-long by 30-foot-wide trestle that provides access to the 100-foot-long by 40-foot-wide loading platform. The trestle has a north-south orientation, and the loading platform an east-west orientation. There are three dolphins east and three dolphins west of the loading platform. These dolphins, connected to the loading platform by catwalks, consist of two breasting dolphins and one mooring dolphin on each side of the loading platform. The structural integrity review conducted for this document by Moffatt & Nichol Engineers included the following:

- Site visit
- Review of existing documents
- Assessment of structural condition
- Assessment of structural capacity

Loading Platform

Petroleum products are offloaded from vessels at the loading platform by two 16-inch diameter and one 12-inch diameter marine loading arms. The loading arms are articulated, tree-like structures capable of handling petroleum products from a range of vessel elevations. A small pipeline manifold area is located inboard of the loading arms. An operator's shelter and emergency switch panel are located inside a metal building on the loading platform. The loading platform has two 100-ton mooring bollards. The loading platform structure consists of a 1-foot-thick slab spanning between 33-inch-deep pile caps. Fifty-one 18-inch square prestressed concrete piles support the structure. Fourteen of the piles are battered in the north-south directions and seventeen of the piles are battered in the east-west direction. The batter piles are inclined at one horizontal to four vertical.

The loading platform appears to be in good condition based on inspection reports and field observations. The Collins Engineers Report (1999) identified a small amount of unsound concrete requiring removal and replacement. The Collins Report also identified approximately 6 percent of the concrete piles as deteriorated. During the Moffatt & Nichol Engineers facility inspection in November 2002, the deck appeared in good condition. Minor repairs recommended in earlier reports, appear to have been completed, as reported by the Moffatt & Nichol engineer.

Dolphins

There are two primary breasting dolphins. These structures resist the majority of the vessel berthing forces. They also provide support to the moored vessel. The west dolphin is located 176 feet west of the center of the loading platform. The east dolphin is located 177 feet east of the center of the loading platform. Each dolphin is 20 feet by

20 feet by 6 feet 5 inches deep and is supported by twenty-two 18-inch square prestressed concrete piles. Eight of the piles are battered in the north-south direction and eight of the piles are battered in the east-west direction. All of the batter piles are battered one horizontal to four vertical. Each dolphin supports a cylindrical rubber marine fender with a large black plastic-faced rub plate against which vessels berth. The fenders and rub plates have been installed within the last two years. Each dolphin also has a 100-ton bollard.

There are two secondary breasting dolphins. These structures provide additional berthing resistance. The west dolphin is located 72 feet west of the center of the loading platform. The east dolphin is located 72 feet east of the center of the loading platform. The dolphins are 16 feet long by 16 feet wide by 5 feet 11 inches deep. Twelve 18-inch-square prestressed concrete piles support each dolphin. Four of the piles are battered in the north-south direction and two of the piles are battered in the east-west direction. All of the batter piles are battered one horizontal to four vertical. The dolphins support cylindrical rubber marine fenders and large black plastic faced rub plates against which vessels berth. The fenders and rub faces have been installed within the last two years. Each dolphin also has a 25-ton cleat.

The primary and secondary breasting dolphins are all in fair to good condition (Collins Engineers 1999). Some hairline cracking in the pile caps was observed by Moffatt & Nichol Engineers. The cracking observed is normal for a 30-year old marine structure and should not affect the dolphins' load carrying capacity. All of the marine fenders are new and in good condition. All of the mooring bollards are in good condition. The Collins report did not identify any deteriorated breasting dolphin piles. It should be noted that Gerwick (2001) indicated that the breasting dolphins are in a deteriorated condition, but did not justify the comment.

There are two mooring dolphins. The mooring dolphins restrain the larger ships' bow and stern lines. The west-mooring dolphin is located 478 feet west of the center of the loading platform. The east-mooring dolphin is located 477 feet east of the center of the loading platform. Each dolphin is 20 feet by 20 feet by 4 feet deep, and is supported by thirty 18-inch-square prestressed concrete piles. Fourteen of the piles are battered in the north-south direction and eight of the piles are battered in the east-west direction. All of the batter piles are battered one horizontal to four vertical. Each dolphin supports a double 100-ton quick release hook with a 3,000-lb pull-in electric capstan and navigational lights.

The mooring dolphins are in fair condition. The east-mooring dolphin has pile cap cracking, epoxied crack repair, and a damaged railing. The west-mooring dolphin has hairline cracks in the cap and concrete delamination under the quick release hook shoe plate. The observed cracking/delamination will not affect the dolphins mooring capacity. The mooring hooks are in good condition. Collins Engineers (1999) identified approximately 3 percent of the concrete piles as deteriorated.

Catwalks

Access to the mooring and breasting dolphins is via timber pile supported "catwalks" from the loading platform. Conduits attached to the catwalk guardrails provide electrical power to the dolphins for the quick release hooks, navigation lights, and other items. The catwalk pile bents are comprised of two timber batter piles, pile cap, and lateral and transverse bracing. The catwalk superstructures consist of two 6- by 16-inch beams, planks, and timber guardrails. Timber piles appear to be from the original construction, and these are battered one horizontal to twelve vertical. There are a total of 40 piles supporting the 20 catwalk bents.

The catwalks are in fair to good condition though piling, braces, and substructure bolting appear to be original, deterioration of these was not noted in the Collins Engineers report (1999). The superstructures, including the longitudinal stringers and deck boards, are generally in good condition, although some of the deck boards at the bents are split. The railings are in good condition.

Trestle and Pipelines

A 1,745-foot-long timber trestle connects the loading platform to the land. The trestle provides a small vehicle lane and support for numerous pipelines including:

- 30-inch dark product
- 12-inch dark product
- 12-inch clean product (2 lines)
- 12-inch vapor recovery
- Fire water, potable water, sewage, electrical, and communication conduits

There are 82 bents in the pipeline trestle with 543 timber piles total. The bents are spaced 23 feet on center. The timber piles are both vertical and battered. There are two types of bents. Most of the bents provide only vertical and transverse lateral support to the pipelines and are supported by six piles each. Four of the piles are battered three horizontal to twelve vertical in the transverse direction, and the remaining two piles are vertical. Every sixth bent is a longitudinal anchor bent supported by eleven piles. Eight of the piles are battered in two directions: two in twelve in the transverse direction, and three in twelve in the longitudinal direction. Two of the piles are battered in the longitudinal direction at three in twelve. The remaining pile is vertical. A pipeline expansion loop is located approximately 980 feet from shore. The piles frame into and are bolted to 12-inch by 12-inch timber bent caps. Timber stringers frame between the bents and provide support for the access roadway and longitudinal continuity.

The construction plans (Hallanger Engineers 1974) do not show the pile lengths or pile capacities. GKO Messinger & Associates (1994a, 1994b), however, indicate that the concrete piles are about 100 feet long, and state: "Pile driving reports developed by Hallanger Engineers in 1974 stated that the (concrete) piles had 45 tons (90 kips) of axial tension capacity. An assumed compression capacity of 60 tons (120 kips) was used for the dolphin capacity analysis." The seismic review work performed by Gerwick in 2001 assigned a precast pile tension capacity of 300 kips (ultimate) and compression capacity of 450 kips (ultimate). The timber piles were assigned ultimate capacities of 20 kips of tension and 33 kips of compression (Gerwick 2001). The length of the timber piles is unknown, but assumed to be about 60 feet.

The 1,745-foot-long timber trestle is generally in good condition, although areas of deterioration exist mostly in the landside untreated piles. These untreated piles were added during the original construction as a contractor convenience and are not required by the plans. Loss of these piles would not result in loss of vertical load carrying capacity of the trestle. Collins Engineers (2001) noted deterioration of only four treated piles, leading to the conclusion that the trestle piles are in good condition despite their age. Bolted connections appeared to be in good condition. The roadway planks and boards have some checks and splits. The bents provide good support to the pipeline. No sags or lack of pipeline support at the bents was observed. Gerwick (2001) found the seismic analysis to be limited in scope and that the trestle bent timber piles were weak.

Gerwick (2001) found that the 30-inch pipeline from the wharf to the trestle is overstrained from lateral movement at each support spaced 23-feet apart along the trestle. While the location of the horizontal restraint of the pipeline on the loading platform is not known in detail, the Gerwick report assumed that the pipeline is restrained about 8-feet from the edge of the trestle. Adding the distance from the edge of the loading platform to the first bent on the trestle, the unsupported length of the pipeline in that area is approximately 20-feet. This raises a concern with regards to the differential movement of the loading platform and the trestle. If it is assumed that the maximum displacement demand for each structure occurs in the opposite direction at the same time, then the pipeline will be overstressed. In addition, about halfway between the loading wharf and the land, the pipelines go through an expansion loop. The behavior of the pipeline/support interface has not been evaluated (Gerwick 2001). Pipeline seismic stresses are thus unknown.

3.11.3 <u>Impacts Analysis and Mitigation Measures</u>

Impact Significance Criteria

Seismic effects could result in significant hazards to structures when not accounted for in design or construction. Impacts are considered adverse and significant if any of the following conditions apply:

- ➢ Settlement of the soil beneath the wharf's foundation that could substantially damage structural components of the wharf.

- ➢ Ground motion due to a seismic event that could induce liquefaction, settlement, or a tsunami that could damage structural components of the wharf.

- Deterioration of structural components of the wharf due to corrosion, weathering, fatigue, or erosion that could reduce structural stability, or such components fail to meet seismic performance requirements.

- Increase in loading conditions, vessel size, or number of vessels that could overstress existing facilities and reduce the structural stability of the wharf.

- Damage to petroleum pipelines and/or valves along the pipeways from any of the above conditions that could release crude/product into the environment.

3.11.3.1 Geotechnical Conditions of the Shore Marine Terminal

Impact GEO-1: Ground Rupture

The Shore facility is not located in the Alquist-Priolo earthquake fault zone. Surface rupture from known active faults is not anticipated, and impacts would be less than significant (Class III).

The wharf and trestle lie outside of the Alquist-Priolo earthquake fault zone and surface rupture from known active faults is not anticipated. Impacts would be less than significant (Class III).

GEO-1: No mitigation is required.

Impact GEO-2: Groundshaking

The impact of berth dredging, natural scour or accumulation of soil in steep slopes near or adjacent to wharf piles should be considered in soil-structure interaction. In addition, liquefaction and lateral spreading resulting from any moderate earthquake may create a significant adverse impact (Class II).

The wharf and trestle are located within a seismically active area with several faults capable of inducing strong ground shaking. Such shaking would result in associated shaking of the structures, including interaction between the soil and structural foundations. Damage to the structures could occur as discussed in detail in Section 3.11.3.2.

The bathymetry in the wharf and trestle vicinity is relatively flat, and lateral spreading of soils at or near the ground surface caused by ground shaking is unlikely. The impact of berth dredging, natural scour or accumulation of soil in steep slopes near or adjacent to wharf piles should be considered in soil-structure interaction. In addition, liquefaction and lateral spreading resulting from any moderate earthquake may create a significant adverse impact (Class II).

Mitigation Measures for GEO-2:

GEO-2a: In the event that such scour has been noted, then Shore shall conduct additional analysis to evaluate the potential for lateral spreading. Loss of lateral support and laterally induced additional loads should be incorporated into the overall analysis and/or design. This analysis should be conducted concurrently with a site specific liquefaction analysis (see Impact GEO-3).

GEO-2b: Seismic evaluation of the structures and their foundations should be included in the structural analysis and geotechnical investigation in compliance with Section 6 of the proposed MOTEMS. The results and recommendations of the evaluation shall be coordinated with the mooring analysis recommendations and implementation of corrections (see GEO-10).

Rationale for mitigation: These studies would determine whether lateral spreading caused by groundshaking would cause any loss of lateral support on the structure. The seismic evaluation would identify any additional corrections that may be needed to ensure structural integrity. Impacts would be reduced to less than significant.

Impact GEO-3: Liquefaction and Seismically Induced Settlement

The site has not had an industry standard liquefaction evaluation performed. As such, the potential for impacts from seismically induced settlement are unknown and this is considered a significant adverse (Class II) impact.

Liquefaction is a phenomenon whereby insufficiently dense saturated granular soil temporarily loses strength and bearing capacity during seismic shaking. If the granular soil is unconfined and on a slope, it tends to spread or flow as mentioned above. Liquefaction usually results in volume reduction that is manifested in ground settlement. Loose, clean sand at relatively shallow depths (low overburden or confining pressures) is most susceptible to liquefaction. Most of the sand from this site appears to be older Pleistocene age sand that is medium dense to dense, based on limited data. As stated in the existing conditions section, the Woodward Lungdren sampling tools and protocols used during the exploration program are outdated and did not include the standard penetration test (SPT), an industry standard for evaluating liquefaction potential. If sand liquefies it could result in volume changes that in turn could result in soil settlement and downdrag on the piles. Because the site does not have an industry standard liquefaction evaluation, the potential for impacts from seismically induced settlement would be considered significant adverse (Class II) impacts.

Mitigation Measures for GEO-3:

GEO-3: Shore shall comply with the proposed MOTEMS. As such, a site specific liquefaction evaluation shall be required to be completed within 6 months after start of the lease. The results and recommendations of the evaluation shall be coordinated with the mooring analysis recommendations and implementation of corrections (see GEO-10).

Rationale for mitigation: The liquefaction evaluation would identify if liquefaction is a problem and would identify corrections to guard against damage to the wharf. Impacts would be reduced to less than significant.

Impact GEO-4: Tsunami

Shore operators may not have adequate warning time to allow a vessel to depart from the wharf to avoid damage to the vessel and/or the wharf from a tsunami. Impacts are considered significant adverse (Class II) impacts.

The maximum expected wave return height near the Shore marine terminal for the 100-year tsunami event would be about 3.3 feet and up to 4.0 feet for the 500-year event. Potential damage to the wharf and/or vessel from these events could occur and impacts are considered significant adverse (Class II) impacts. As tsunamis can be generated either by a distant or near source, the Shore operators may or may not have adequate warning time for which to allow the vessel to depart from the wharf to avoid damage.

Mitigation Measures for GEO-4:

GEO-4a: As soon as possible, after notification of a tsunami, Shore operators shall release the vessel from its mooring and the vessel shall move away from the wharf.

GEO-4b: Shore shall comply with Section 5 of the proposed MOTEMS mooring analysis (see GEO-10).

Rationale for mitigation: Adherence to the mitigation measures would reduce, to the maximum extent feasible, the potential for damage that could occur to a vessel and/or the wharf during a tsunami. Impacts would be reduced to less than significant.

3.11.3.2 Structural Conditions of the Shore Marine Terminal

Seismic Performance Standards

The proposed MOTEMS (California State Lands Commission 2004) requires that a marine oil terminal facility satisfy Level 1 and Level 2 seismic performance criteria. The seismic performance criteria depend on the predicted maximum earthquake motions at the site and the potential size of an oil spill. Seismic performance criteria are expressed in terms of seismic acceleration with a given probability of occurrence. The criteria for this site are as follows:

- Level 1 Seismic Performance (50 percent probability of occurrence in 50 years or the earthquake with a 72-year return period):
  - Minor or no structural damage

- Level 2 Seismic Performance (10 percent probability of occurrence in 50 years or the earthquake with a 475-year return period):
  - Controlled inelastic structural behavior with repairable damage
  - Prevention of structural collapse
  - Temporary loss of operations, restorable within months
  - Prevention of major spill (> 1,200 barrels)

Impact GEO-5: Loading Platform Structural Adequacy

During a Level 2 seismic event, the batter piles are expected to behave in a nonlinear fashion. The loading platform would undergo significant softening as a result of the global nonlinear behavior. Impacts are less than significant (Class III).

The seismic evaluation report (Gerwick 2001) formed the basis for the anticipated performance of the loading platform, breasting dolphins, mooring dolphins, and pipeline trestle. The seismic evaluation report checked structure adequacy for Level 1 and Level 2 earthquakes.

The anticipated Level 1 and Level 2 seismic displacements of the loading platform are 0.16 feet and 0.29 feet, respectively. The failure mode is pile pullout, which is a gradual and desirable response. Batter pile pullout may begin to occur only as a result of Level 1 seismic loading. During a Level 2 seismic event, the batter piles are expected to behave in a nonlinear fashion. The loading platform would undergo significant softening as a result of the global nonlinear behavior. However, structural collapse is not expected to occur as a result of the Level 2 earthquake. Impacts are less than significant (Class III).

<u>GEO-5: No mitigation is required</u>.

Impact GEO-6: Dolphins Structural Adequacy

If secondary breasting dolphins are not upgraded, the potential for significant adverse impacts from an oil spill is small (Class III).

The mooring dolphins and primary breasting dolphins were found to be adequate for Level 1 and Level 2 earthquakes (Gerwick 2001) and impacts are expected to be less than significant (Class III).

However, greater seismic displacements are predicted for the secondary breasting dolphins due to the fewer number of batter piles and greater mass. Gerwick (2001) recommended evaluation due to the piles' deteriorated condition and, anticipated seismic displacements. Since the Gerwick report, Shore has replaced the fenders and made some concrete repairs and relatively little deterioration was observed on the Moffatt & Nichol Engineers field visit in November 2002. Seismic damage could occur if secondary breasting dolphins have not been adequately upgraded, potentially shutting down wharf operations until necessary repairs are made. Preventative maintenance to ensure the adequacy of the secondary breasting dolphins is highly recommended.

GEO-6: No mitigation is required.

Impact GEO-7: Catwalks Structural Adequacy

Damage to catwalks from a seismic event would not result in an oil spill, and damage can easily be repaired. Impacts are less than significant (Class III).

The seismic performance of the catwalks has not been evaluated. However, their performance is of secondary importance, because a catwalk failure will not result in an oil spill and can easily be repaired. Impacts are expected to be less than significant (Class III).

GEO-7: No mitigation is required.

Impact GEO-8: Trestle Structural Adequacy – Batter Pile to Bent Cap Connections

During an earthquake damage could occur in the batter pile to bent cap connections and could damage the trestle. This would result in a significant adverse impact (Class II).

As previously described, there are two types of trestle bents. The majority of the trestle bents are two-dimensional structures designed to resist only vertical and transverse forces. During an earthquake high forces develop in the two 1-1/2 inch bolts at the batter pile to bent cap connections. It appears probable that these connections do not have the capacity to transfer the calculated forces and significant adverse impacts (Class II) could result.

Mitigation Measures for GEO-8:

GEO-8: Within one year of the new lease, Shore shall reevaluate the loads on the bents, check the batter pile bolt connections, and adopt corrective mitigation measures.

Rationale for mitigation: Implementation of the mitigation provides a reevaluation of the batter pile bolt connections and provides the means for correction. With implementation of the mitigation measure, the impact would be reduced to less than significant.

Impact GEO-9: Trestle Structural Adequacy – Anchor Bents

The anchor bent batter pile to bent cap bolts are not capable of transmitting the predicted transverse seismic loads that could result in a loss of support for the petroleum pipelines and a spill could occur. This would result in a significant adverse impact (Class II).

The second type of trestle bents are anchor bents, of which there are twelve. The anchor bent batter pile to bent cap bolts are not capable of transmitting the predicted transverse seismic loads. One 1-inch bolt transmits the full tension load from two batter piles into the 12-inch by 12-inch bent cap. The ultimate bolt capacity is less than 10 kips, while the demand, based on maximum pile tension, is roughly 40 kips. The loads indicate that these connections will fail during an earthquake resulting in a significant adverse impact (Class II). The bolted connection in the anchor pile bents could result in loss of support for the petroleum lines and potentially initiate an oil spill. Some of these pipelines contain petroleum products at all times (they are not "stripped" following fuel handling), and structural failure of the trestle could result in an oil spill of up to 1,500 barrels (Gerwick 2001).

Mitigation Measures for GEO-9:

GEO-9: Shore shall reevaluate the loads in the anchor bents and batter pile connections within one year of the new lease. The anchor bents' inadequacy should be addressed and corrective measures implemented within 2 years.

Rationale for mitigation: Implementation of the mitigation would assure that the anchor bent batter pile to bent cap bolts can transmit the predicted seismic loads such that there would be no support loss of the petroleum pipelines. This would reduce the risk of an oil spill due to broken pipelines. The impact would be reduced to less than significant.

Impact GEO-10: Berthing/Mooring Load Capacity

The last mooring analysis used data from sites nearby that may not reflect actual wharf conditions. There could be potentially significant direct and indirect impacts (Class II) associated with berthing and mooring capacity under actual currents, tides, and winds, with the potential for oil releases.

GKO Messinger & Associates (1994) indicates that there are significant berthing and mooring limitations for large vessels in order to limit the load to the dolphins. These limitations restrict the load on the dolphins to the pile allowable capacities. Based on these limitations, berthing and mooring forces are less onerous than the seismic loading conditions. The report performed was a structural appraisal and not a detailed mooring analysis. As no mooring analysis in compliance with the proposed MOTEMS has been performed for the Shore marine terminal, and since there could be potential direct and indirect impacts associated with berthing and mooring stresses on the facility, with potential for oil releases if an accident were to occur, impacts are potentially significant adverse impacts (Class II).

Mitigation Measures for GEO-10:

GEO-10a: Shore shall collect 12 months of data on currents, tide levels, and wind speed/direction at the wharf.

GEO-10b: If data analysis shows that currents, tides and wind speeds are significantly different (as assessed by CSLC) from that assumed in the previous analysis, Shore shall conduct a new mooring analysis consistent with the proposed MOTEMS Section 5 requirements within 12 months.

GEO-10c: Within 12 months of the start of the new lease, Shore shall conduct a passing vessel study for vessels navigating within 500 feet of the wharf, as per MOTEMS requirements.

Rationale for mitigation: The mitigation measures would determine if the existing mooring system on the wharf is in compliance with the proposed MOTEMS requirements, and would identify any needed corrections. With implementation of the corrections the potential for damage to both the wharf and vessels would be reduced to less than significant.

Impact GEO-11: Pipelines

Pipeline stresses on the 30-inch pipeline in relation to movement of the loading platform and trestle, and on the pipeline expansion loop support interface along the trestle are unknown. The potential may exist for damage to the pipeline and oil leakages that would result in a significant adverse impact (Class II).

Gerwick (2001) found concern with regards to the 30-inch pipeline and differential movement of the loading platform and the trestle. If it is assumed that the maximum displacement demand for each structure occurs in the opposite direction at the same time, then the pipeline will be overstressed. In addition, about halfway between the loading wharf and the land, the pipelines go through an expansion loop. The behavior of the pipeline/support interface has not been evaluated (Gerwick 2001), and thus, the pipeline seismic stresses at this interface are unknown. A significant adverse impact (Class II) results, as pipelines could be stressed to the point where damage and leaks could result.

Mitigation Measures for GEO-11:

GEO-11a: Within 6 months of the start of the lease, Shore shall conduct a pipeline analysis on the 30-inch pipeline and the pipeline loop.

GEO-11b: Shore shall ensure that pipelines for oil transfer meet MOTEMS and CSLC regulations in CCR Title 2, Division 3, Chapter 1, Article 5.5, Sections 2564 through 2570 for ensuring pipeline integrity.

Rationale for mitigation: The pipeline analysis would determine the need for corrections associated with the 30-inch pipeline and the pipeline loop. Corrections would reduce the potential for oil spills. Ensuring pipeline integrity in compliance with regulations also reduces the potential for leaks or spills of oil. With implementation of the mitigation measures would be reduced to less than significant.

3.11.4 Alternatives

3.11.4.1 No Project Alternative

Impact GEO-12: Effects on Geotechnical Issues with No New Shore Terminals Lease

The geotechnical and structural impacts of the wharf as described for the Proposed Project, would not be of concern with no new lease. As compared to the Proposed Project a beneficial (Class IV) impact would result. Similar conditions of the Proposed Project may occur at the other marine terminals that would increase operations as a result of no new lease for Shore Terminals. Shore Terminals has no responsibility at those facilities.

The No Project Alternative would require Shore to cease operation of the marine terminal, which currently serves nearby refineries between Rodeo and Martinez. Thus, the potential concerns as to geological issues and structural conditions of the wharf as described for the Proposed Project, would not be of concern with no new lease. Decommissioning the wharf would require a separate CEQA review. Deconstruction activities would not result in any geotechnical impacts.

Without the Shore marine terminal, other area marine terminals would be required to increase inbound and outbound shipments to meet regional refining demands. Increasing the number of shipments at the other area marine terminals should not result in geotechnical impacts since these wharves are operational. Any activity associated with a wharf accepting larger vessels than the wharf is currently able to handle, may be subject to a separate CEQA review and structural evaluation.

GEO-12: No mitigation is required.

3.11.4.2 Increased Use of Existing Pipelines for Continued Operation of Upland Facility Alternative

Impact GEO-13: Continued Shore Upland Operations via Existing Pipelines

Seismic forces and pipeline age and integrity problems can result in leaks and spills, resulting in a significant adverse (Class II) impact.

For this alternative, it is assumed that the Shore upland facility would continue to function utilizing only land-based pipelines. Connections for moving oil to and from the Shore upland facility to the Shell Refining Martinez, Valero Benicia, and Tesoro Amorco wharves are already in place. Therefore, no construction would be required to use these pipelines and no geotechnical construction impacts would occur. Leaks and spils from pipelines can be caused by seismic forces, improper engineering design, corrosion, and joint failure, and have the potential to result in significant adverse impacts (Class II), which would then impact other resources as presented in other sections of this EIR. In comparison to the Proposed Project, the impacts for this alternative would be greater for land based pipeline risks, but less overall in terms of spill potential.

Under this alternative, increased shipping would occur at these wharves, and no impacts are foreseen. Any activity associated with a wharf accepting larger vessels than the wharf is currently able to handle, may be subject to a separate CEQA review and structural evaluation.

Mitigation Measures for GEO-13:

GEO-13: Damage to pipelines by seismic forces and other hazards can be minimized by providing proper engineering design, periodic inspection, maintenance, and retrofitting of pipelines to reduce the possibility of pipeline failure due to corrosion and fatigue. Shore Terminals shall apply this measure to any pipelines for which Shore owns or has responsibility.

Rationale for mitigation: By providing proper engineering, inspection, maintenance and retrofitting, the potential for pipeline failure can be reduced to less than significant.

3.11.4.3 Modification of Existing Pipelines for Continued Operation of Upland Facility Alternative

Impact GEO-14: Continued Shore Upland Operations via Modifications to Existing Pipelines

Use of the PG&E line would require examination of pipeline integrity and construction of missing segments and connections. No significant adverse construction impacts (Class III) would be expected. Seismic forces and pipeline age and integrity problems for PG&E and other lines could result in leaks and spills, resulting in a significant adverse (Class II) impact.

Shore has connections to the inactive PG&E fuel oil line that could transfer crude oil to and from Shore with possible connections to Shore Selby, ConocoPhillips Rodeo, and Chevron Richmond. To use this line would require examination of pipeline integrity, construction to reconnect the segment in the city of Martinez, and construction to provide connections to these three marine terminals. A geologic evaluation would be required to determine if any potential geotechnical issues could occur along the pipeline route. There could be a potential for seismic impacts. Pipelines are typically flexible enough to withstand strong ground shaking without rupturing. Special design or flexible connections need to be considered for areas where the pipeline crosses active faults and at connecting points to valves and storage facilities. However, leaks and spills from pipelines can be caused by seismic forces, improper engineering design, corrosion, and joint failure, and have the potential to result in significant adverse impacts (Class II), which would then impact other resources as presented in other sections of this EIR. In comparison to the Proposed Project, the impacts for this alternative would be greater due to both construction and risk of leakage along land-based pipelines, but less overall in terms of spill potential.

Under this alternative, increased shipping would occur at these three wharves, and no impacts are foreseen. Any activity associated with a wharf accepting larger vessels than the wharf is currently able to handle, may be subject to a separate CEQA review and structural evaluation.

Mitigation Measures for GEO-14:

GEO-14: Damage to pipelines by seismic forces and other hazards can be minimized by evaluating geotechnical hazards and providing proper engineering design for new pipeline alignments. For all pipelines, periodic inspection, maintenance, and retrofitting of pipelines should be conducted to reduce the possibility of pipeline failure due to corrosion and fatigue. Shore Terminals shall apply this measure to any pipelines for which Shore owns or has responsibility.

Rationale for mitigation: By providing proper engineering, inspection, maintenance and retrofitting, the potential for pipeline failure can be reduced to less than significant.